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AN EVALUATION OF THE LOW TEMPERATURE PERFORMANCE OF RECYCLED HOT MIX

MATERIALS INFORMATION

ENGINEERING MATERIALS OFFICE

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AN EVALUATION

OF

THE LOW TEMPERATURE PERFORMANCE

OF

RECYCLED HOT MIX

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AN EVALUATION OF THE LOW TEMPERATURE PERFORMANCE OF RECYCLED HOT MIX

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Abstract:

The Ministry of Transportation, Ontario embarked on a recycled hot mix (RHM) program in 1979. Early observations of mixes placed within the first three years of the program indicated that RHM needed improvement particularly with respect to cracking. As a result, a long term monitoring program was planned. This paper presents the results of an evaluation on the ability of RHM to resist thermal cracking. The evaluation was undertaken as part of the long term program.

The resistance to low temperature cracking has been investigated in terms of both limiting stiffness and fracture temperature criteria. In the limiting stiffness approach, McLeod's tentative design guide was used to evaluate the resistance of the mixes to thermal cracking. In the case of fracture temperature criterion, the failure strains under uniaxial direct tension tests and the induced thermal strains were used to estimate the fracture temperature of asphalt mixes. On the basis of both criteria, the test results confirm the common belief that recycled hot mixes are more susceptible to thermal cracking than conventional/virgin hot mixes. Also, when the results of both criteria are compared with observed field data and recovered penetration values, the fracture temperature criterion appears to be more suitable than the stiffness criterion for evaluation of low temperature performance.

In order to reduce the susceptibility of pavements to thermal cracking, it is recommended that penetration values of RHM be improved by either reducing the recycling ratio and/or use of suitable grades of rejuvenating/virgin asphalt cement. Additionally, the fracture temperature of RHM should be determined during the mix design stage to ensure compliance to expected winter temperature.

Abrégé:

Le ministère des Transports de l'Ontario a mis sur pied un programme d'étude des enrobés à chaud recyclés en 1979. L'observation des enrobés utilisés au cours des trois premières années du programme a révélé que leur qualité devait être améliorée surtout en ce qui concerne la fissuration. Un programme de contrôle à long terme a donc été établi. Ce rapport présente les résultats d'une évaluation du potentiel de résistance des enrobés à la fissuration thermique. Cette évaluation a été effectuée dans le cadre du programme à long terme.

La résistance à la fissuration à basse température a été étudiée selon les critères de rigidité limite et de cassure thermique. Dans l'approache de la rigidité limite, le guide de conception expérimentale de M. McLeod a été utilisé pour évaluer la résistance des enrobés à la fissuration thermique. Dans le cas du critère de cassure thermique, on a comparé les tensions de rupture pratiquées lors d'essais de traction directe uniaxiale à la tension thermique induite afin d'évaluer la température de cassure des mélanges bitumineux. Les résultats des essais effectués selon les deux critères ont confirmé la croyance commune selon laquelle les enrobés à chaud recyclés sont plus susceptibles à la fissuration thermique que les enrobés à chaud purs traditionnels. En outre, les conslusions tirées des résultats fondés sur les deux critères ont éte comparés aux données de terrain et à la pénétrablilité obtenue et il ressort que le critère de cassure thermique convient mieux que celui de la rigidité.

Afin de réduire la susceptibilité des revêtements à la fissuration thermique, on recommande d'améliorer la pénétrabilité des enrobés soit en réduisant le ratio des matériaux recyclés ou en utilisant les catégories convenables de bitume régéné et pur. De plus, la température de fissuration des enrobés à chaud recyclés devrait être déterminée au cours de la phase de conception des enrobés afin d'assurer qu'ils sont conformes à la température hivernale prévue.

Keywords: Asphalt, recycled hot mix, low temperature performance, limiting stiffness, fracture temperature, induced strain, thermal cracking, field data.

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1. INTRODUCTION

The Ministry of Transportation of Ontario embarked on a hot mix recycling program in 1979. It was based on the recommendations presented by a task force set up to study the feasibility of using recycled hot mixes as a viable alternative for highway pavement rehabilitation projects. The task force also recommended an implementation package which called for an immediate commitment of substantial tonnages in the construction programs to encourage capital investment on equipment by paving contractors for recycling of hot mixes.

By 1980, some half of a million tonnes of recycled hot mix (RHM) were constructed, representing about 20% of the annual paving program [1]. Early observations of the performance of these mixes suggested that RHM needs improvement, particularly with respect to the resistance to thermal cracking. This created a need for an objective evaluation of RHM, both in the laboratory as well as long term monitoring in the field. An experimental program was therefore initiated by carrying out a series of laboratory tests to evaluate the properties of RHM. This paper presents the findings on the part of study on low temperature performance of RHM placed during the first few years of the program.

2. SCOPE OF EVALUATION

The basic scope of the study is to investigate the relative resistance of recycled hot mixes to thermal cracking, as compared to conventional hot mix. In more specific terms, the objectives can be stated as follows:

(1) To examine the existing method of approach and select the appropriate methodology for evaluating the low temperature susceptibility of RHM;

(2) To design an experimental program to determine the criteria for the methodology selected in step 1;

(3) To analyse the test results and compare the low temperature performance of various RHM with a conventional hot mix;

(4) To determine if laboratory mixed samples can be used for low temperature evaluation instead of plant mixed samples;

(5) To identify the best method for a future low temperature evaluation study based on laboratory and field results;

(6) To suggest improvement, if any, for future mix design procedures.

3. EVALUATION METHOD

A number of procedures have been developed for evaluating low temperature susceptibility of bituminous materials and they can be categorized as follows:

(a) use of asphalt binder rheology,
(b) use of limiting stiffness,
(c) predicting fracture temperature, and
(d) estimating cracking frequency.

Asphalt binder rheology has been considered a controlling factor in cold temperature cracking. However, its use is not applicable to this case because it deals with only the binder and not the overall performance of asphaltic mixes. The approach of using limiting stiffnesses has evolved, as a result of field observations and much of the laboratory work [2 & 3]. The method involves the determination of mix stiffness and compares such values with an established design guide [4]. If the estimated value exceeds the design value, cracking is likely to occur.

The prediction of fracture temperature is based on the thermally induced stresses or strains in the bituminous layer. When the induced stress or strain, due to temperature drop, exceeds the failure stress or strain, cracking is expected to occur. The corresponding temperature is called the fracture temperature. The higher the fracture temperature of a material, the lower is its resistance to thermal cracking. In a design situation, if the estimated fracture temperature is less than the winter design temperature [5], cracking is expected to occur and the design should be rejected or modified.

The approach of estimating cracking frequency developed by Haas [5] is based on the stiffness of asphalt cement, and not the asphaltic mix. As such, this method is not considered in this investigation.

In summary, the method of evaluation for this study is confined to two criteria: limiting stiffness and fracture temperature.

4. LABORATORY EXPERIMENT

A laboratory test program was designed specifically to investigate the potential of different asphaltic mixes to resist thermal cracking, using limiting stiffness and fracture

temperature criteria. The program consists of a series of test, performed on a conventional hot mix (HL 4) and different RHM specimens, to establish performance indicators for the mixes. The type of tests included in the program are:

(a) direct tension tests at various temperatures, and
(b) thermal contraction tests.

A detailed description of theories, applications and interpretation of results of the direct tension test can be found in reference [6].

4.1 Material Selection

Materials were selected from five recycling Contracts (81-74, 81-224, 82-04, 82-19 and 83-43), which cover different regions, virgin asphalt cements (A.C.) and recycling ratios. The locations and traffic volumes of different RHM sites selected are shown in Table 1. The recovered asphalt binder and aggregate properties are given in Tables 2 and 3.

For the RHM, specimens were produced from plant mixes and individual mix components in the laboratory. However, for Contracts 81-74 and 82-04, the A.C. used for making laboratory mixes was different from that for the plant mixes, because of non-availability of the binders as used in the field. Penetration grade asphalt cements, ranging from 85/100 to 300/400, were used for the RHM.

A crushed limestone aggregate and an 85/100 penetration grade A.C. were used for making conventional mix (HL 4) specimens. The material properties are given in Tables 2 and 3.

4.2 Mix Designs

The RHM mix designs cover different recycling ratios, ranging from 25/75 to 70/30. They were designed to meet the usual Ministry specification requirements. Except for Contract 83-43 where the mix was designed using an HL 1 RAP material from a surface course of a freeway, the others were HL 4 mixes for secondary highways. The HL 1 RAP material consisted a good quality steel slag coarse aggregate and natural sand fine aggregate. The mix design properties are shown in Table 4 and mix gradations in Table 5.

Due to the use of various recycling ratios and RAP with low recovered penetrations, virgin A.C. of different penetrations were used to bring the overall penetration of the RHM to desirable levels, which are normally obtainable for virgin mixes.

4.3 Sample Preparation

Test samples were prepared according to procedures given in ASTM D3202, using the California kneading compactor. Compacted beam specimens of length 381 mm, depth 88 mm and width 82 mm were saw cut into three shorter sections of 100 mm in length and 76 X 76 mm^2 in cross-section. For the direct tension test, each specimen was glued to steel end-plates with epoxy, and allowed to cure overnight before temperature conditioning and testing. For thermal contraction measurements, the specimens were used as cut without further additional treatment.

4.4 Direct Tension Test

Direct tension tests were performed, using a MTS servo hydraulic machine at 21°C, -5°C and -35°C, to determine the tensile strengths, strains, and stiffnesses of different mixes. The factorial arrangement for the testing program is shown in Table 6. The specimen was pulled at a constant rate of extension (3×10^{-2} mm/min for tests at -5°C and -35°C, and 5 mm/min at 21°C) until failure occurred. The stress, strain and stiffness at failure were determined (see Tables 7 and 8). For testing at temperatures -5°C and -35°C, the specimens were pre-cooled overnight prior to testing. An environmental chamber was used to maintain the low temperatures of -5°C and -35°C.

4.5 Thermal Contraction

Determination of thermal contraction is an integral part of the analysis of the thermal behaviour of asphalt mixes. Thermal contraction was used to estimate the induced strain due to thermal shrinkage under restraint condition. Set points were selected on each sample and the lengths were measured using a digital calliper before and after cooling the specimens for 24 hours at each temperature. Readings were taken during both the cooling and heating cycle for comparison. Thermally induced strains at various temperatures are calculated for all mixes by dividing the actual contraction of each specimen by its original length (Tables 9 and 10).

5. RESULTS

Analysis of the low temperature cracking properties of the mixes is made in terms of both limiting mix stiffness and pavement fracture temperature criteria. Also, the results obtained from plant mixes are compared with those from laboratory mixes. All of these results are then, in turn,

compared with the HL 4 mix and with field data and expected trends, using the recovered penetration values of the asphalt cements used.

5.1 Limiting Mix Stiffness Criterion

In general, the mix stiffness increases with decrease in temperature (Figs. 1 & 2). McLeod's tentative design guide [4] has been used for the limiting stiffness analysis. The stiffnesses of RHM and the conventional mix at various temperatures are compared with McLeod's maximum criterion. It appears that none of the mixes has resistance to cracking at low temperature (Figs. 1 and 2). However, it is noted that McLeod's values for limiting stiffness, according to the Asphalt Institute Research Report [5], may be too conservative because of its dependency on the PVN values, which tend to give lower stiffnesses than the actual measured values. Additional safety factors could have been built into such procedures to produce these limiting values. This assumption is confirmed by comparihg the McLeod's maximum stiffness values at which cracking is expected to occur with the measured values at the fracture temperatures. Results show that a range of factors of about 2 at -23°C and 7 at -13°C existed when dividing the stiffness values at fracture temperatures by McLeod's values (Table 8). The stiffness values at fracture temperature were obtained from the graphs in Figs. 1 and 2 at the fracture temperatures shown in Figs. 5 and 6 for the different mixes.

5.1.1 Relationship of Plant and Laboratory Mix Stiffnesses

One of the objectives of this study has been to determine if laboratory mixed samples can be used, instead of plant mixed samples, for evaluation of the performance of RHM at low temperature. This point is addressed by comparing the stiffnesses of mixes produced in the plant and in the laboratory (Fig. 3). Laboratory mixed specimens exhibit higher stiffness values (ranging from 21% to 54%), at warmer temperatures i.e. 21°C, than the plant mixed specimens, and lower stiffness values (ranging from 38% to 142%, except for Contract 81-74), at very low temperatures i.e. -35°C. From these results, it is clear that the stiffness value at any given temperature is not sufficient to adequately evaluate the behaviour of asphaltic mixes at low temperatures. The percentage difference between laboratory and plant mix stiffnesses appears significant, averaging to about 30% for both ends of

the temperature range (Table 8), when the stiffness value for the mix from Contract 82-04 was excluded. However, a plot of the logarithmic values (Fig. 4) shows excellent correlation ($r^2 = 0.976$) between the plant and laboratory mix stiffnesses. The difference in stiffness, between the plant and laboratory mixed specimens, may be due to the different degrees of binder hardening as a result of the different mixing and production procedures used for preparing the mixes.

Based on the good correlation between the stiffness values obtained from the laboratory and plant mixes, future low temperature stiffness evaluation of RHM can reliably use samples that are prepared in the laboratory.

5.1.2 Comparison of Virgin Mix and RHM

The RHM from all contracts show higher stiffness values than the HL 4 mix. This confirms the general perception that virgin mixes will perform better than RHM with respect to thermal cracking.

5.2 Fracture Temperature Criterion

Fracture temperature (F.T.) of a pavement is defined as the temperature at which the pavement cracks due to shrinkage. It could be estimated by plotting the failure strains (Table 7), determined from direct tension tests and the induced thermal strains (Tables 9 and 10) of different hot mixes against decreasing temperature, as shown in Figs. 5 and 6. The temperature at which the induced thermal strain exceeds the failure strain corresponds to the F.T. of the mix. These estimated fracture temperatures are shown in Table 7.

5.2.1 Relationship of Plant and Laboratory Mix Fracture Temperature

Laboratory mixed samples generally have higher F.T. than plant mixes (Figs. 7 & 8). This indicates that laboratory samples have a lower resistance to thermal cracking than plant samples. The trend is based on the results of the three Contracts shown in Fig. 7, and excluding the value for Contract 82-04 because of its much harder binder properties (Table 2), as will be discussed later. As mentioned previously under 5.1.1, the difference could be associated with the different mix production procedures. However, as can be seen in Fig. 7, there is a clear relationship between the F.T. for

the plant and laboratory mixes. In this respect, it is possible to use laboratory mix to evaluate field performance, once the trend is established or substantiated by more data.

The lower resistance to thermal cracking of the laboratory mixes is only in agreement with the results of stiffness analysis at 21°C, and not the -35°C values (Table 8). Since the intent of this evaluation is to determine the low temperature thermal cracking properties of asphalt mixes, stiffness measurement at -35°C is considered more appropriately related to the F.T. results than those of the 21°C. Also, as stiffness value at any given temperature is not sufficient to evaluate low temperature performance, and as F.T. is a more direct measurement of the fracture property of a mixture, F.T. approach should therefore be more reliable for predicting low temperature performance.

5.2.2 Comparison of Virgin Mix and RHM

Except for Contract 82-18, all other mixes had F.T. above the -23°C F.T. of the virgin HL 4 mix. This again confirms the previous findings, based on the stiffness modulus, and the general belief that RHM is more susceptible to thermal cracking than virgin mixes.

5.3 Effect of Binder Properties on Stiffness and Fracture Temperature

As fracture characteristics of asphalt mixes are more affected by the properties of the binder used, the relationships of the asphalt cement penetration and viscosity and mix stiffness and fracture temperature are therefore examined.

It appears that there is a general trend, showing that mix stiffness at fracture temperatures increases with an increase in penetration and a decrease in viscosity values (Fig. 9a & 9b). This means that a mix, when tested at low temperature for cracking, can achieve a high stiffness value above the limit as suggested by McLeod, provided the binder in the mix does not fracture before reaching the McLeod's limits.

Fig. 9a also shows that two curves could be drawn for mixes with the high and the low penetration values. The lower penetrations were caused by a combination of factors, such as initial low RAP recovered penetration value, low percentage of the added virgin asphalt cement and mixing in a drum plant without a centre feed.

A good relationship is obtained when mix fracture temperature is correlated with the results of recovered asphalt cement penetration and viscosity of the RHM (Figs. 10a & 10b). On the basis of these data, mixes with recovered penetration values of 70 to 80, or viscosity of 200 to 300 mm^2/s, can be expected to be able to resist cracking to as low a temperature as -30°C. These graphs further confirm the common belief that penetration values of about 60 is sufficient for Southern Ontario, and values greater than 70 are necessary for Northern Ontario, for resisting low temperature pavement cracking.

5.4 Field Observation

Field condition surveys were carried out including crack mapping of different types of crack as shown in Table 11. The crack surveys data, in terms of metres of cracks per lane kilometre, is compared with the laboratory results. Contracts 82-19, 81-224 and 81-74 have total pavement crack lengths between 100-400 m as against to the lengths of about 3,000 m for Contracts 82-04 and 83-43. This indicates that the RHM from the former three Contracts are less susceptible to cracking than from Contracts 82-04 and 83-43. This is in apparent agreement with the findings so far obtained, particularly in respect to the higher F.T. and lower recovered penetration values obtained for the latter two contracts.

When the relationships of metres of cracks per lane kilometre and F.T. and stiffness are being examined respectively, a good correlation of $r^2 = 0.913$ for F.T. (Fig.11a) and a poor correlation of $r^2 = 0.484$ for stiffness (Fig. 11b) are obtained. These relationships confirm the better use F.T. than stiffness criteria for evaluating low temperature cracking. Additionally, similar to Fig. 10, the relationship between crack lengths and recovered RHM penetration is better than the one between crack lengths and viscosity (Fig. 12). This finding indicates that the common approach of using penetration values at 25°C, rather than viscosities at 135°C, for selecting the A.C. grades in mix designs for low temperature performance is valid.

5.5 Performance Rating of Mixtures

For the purpose of evaluating the relative resistance of the mixes to thermal cracking, a rating system of from 1 to 5, (correspond to very good to poor resistance) is used. The mixes are rated by their stiffness values, fracture temperatures and field data (Table 10). Contract 82-19 and conventional HL 4 mixes

show consistently better ratings than the others, except when they were rated under the stiffness criteria at -35°C. The other mixes from Contracts 82-04 and 83-43 were rated poor for causes as discussed in 5.3. The rating is consistent with the field results shown in 5.4. The RHM (Contracts 82-19, 81-224, 81-74) that performed well are those mixes with recycling ratios at or below 60/40, 50% or more 300/400 penetration grade virgin asphalt cement, and recovered penetration values similar to conventional mixes, such as the HL 4 mix used in this study.

6. PREDICTION METHODS FOR LOW TEMPERATURE PERFORMANCE

The two approaches, McLeod's limiting stiffness criteria and the fracture temperature, are further examined below in terms of both field data and material standards e.g. penetration values, as to their suitability for predicting low temperature performance of hot mixes:

(a) Appropriateness of limiting stiffness -- previous discussion (in 5.1.1 and 5.2.1) stated that McLeod's values are considered conservative and inadequate for predicting low temperature behaviours of hot mixes. It is evident in Fig. 11 that McLeod's limit is well below the fracture stiffness values obtained for this study. McLeod's values may not be just conservative, because a family of limits could exist for different types of mixes e.g. rich, medium and harsh/dry mixes. Possibly, McLeod's data was for the early richer and finer asphaltic mixes of higher binder content type (e.g. 6%). The results of the "medium" mix type, represented by the three good mixes used in this study, are plotted as a solid line in Fig. 13. A good correlation with r^2 = 0.997 is obtained. The dotted line is for the "dry" mix type, similar to the mixes used in Contracts 82-04 and 83-43. If this argument holds true, the limiting stiffness method will have to be further examined and modified accordingly before it can be used with confidence.

(b) Appropriateness of fracture temperature -- the findings so far obtained reveal that the fracture temperature approach is more suitable, for evaluating low temperature performance of hot mixes, for the following reasons:

* It has been substantiated by the good correlation with field data.
* It correlates well with recovered penetration values and follows the expected trend of behaviours

i.e. higher penetration corresponds to lower fracture temperature.

* It is a direct measurement and can be compared with expected local/winter temperature of a given pavement, to ensure that a mix to be constructed has adequate low F.T. properties.

In general, despite the common practice of using factors such as penetration (including penetration index and pen-visc number), resilient modulus and viscosity gradients etc. to evaluate low temperature performance of asphaltic mixes, it appears that, once the fracture temperature is known, all other information becomes less important.

7. CONCLUSIONS

(i) Materials Performance

(a) The findings confirmed the general belief that recycled hot mixes are more susceptible to thermal cracking than conventional hot mixes.

(b) Based on the fracture temperature results, recycled hot mixes from Contracts 81-74, 82-19 and 82-224 have better resistance to cracking than those from Contracts 82-04 and 83-43.

(c) Both field data and laboratory results reveal that hot mixes recycled at low recycling ratios (e.g. 30/70 as in Contract 82-19), or using high penetration virgin asphalt cement (e.g. 300/400 as in Contracts 81-74 and 81-224), have better performance than those with high recycling ratios (e.g. 70/30 as in Contract 82-04) or using low penetration virgin asphalt cement (e.g. 85/100 as in Contract 83-43).

(d) The findings confirm that the use of higher penetration grade asphalt cement (e.g. 150/200) in Northern Ontario is appropriate, because it has an expected F.T. of below -35°C.

(ii) Evaluation Methods

(e) Fracture temperature method is considered more suitable than limiting stiffness criteria for evaluating the potential of mixes to resist low temperature cracking.

(f) Fracture temperature ratings of mixes used in this study correlate well with ratings based on field data.

(g) Fracture temperatures also correlate well with recovered penetration values. The relationship supports the common belief that higher penetration mixes tend to have lower fracture temperatures.

8. RECOMMENDATIONS

The following recommendations are made, to minimize low temperature cracking and improve the accuracy of predicting fracture temperature:

(a) limit recycling ratio to a maximum of 50/50,

(b) select the appropriate virgin asphalt cement to produce desirable final/recovered mix penetrations,

(c) evaluate the mix fracture temperature to ensure compliance with expected winter temperature,

(d) determine experimentally the actual rate of contraction of the specimen, at a specific rate of cooling representative of field conditions, and use the same rate to extend the specimen in the direct tension test,

(e) carry out additional direct tension tests at -15°C and -25°C to obtain more data in the low temperature region,

(f) further substantiate the possibility of a family of curves for stiffness and fracture temperature for the rich, medium and dry mixes, and

(g) obtain more data to support the use of laboratory mixed samples for prediction/evaluation of low temperature performance in the field, for both RHM and conventional mixes.

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Table 1: Location and Traffic Volume (AADT) for Different RHM Monitoring Sections

CONTRACT	REGION	HWY	LOCATION	YEAR					
				81	82	83	84	85	86
81-74	EASTERN	60	35km E. of Renfrew	1500	1500	1500	1350	1350	1450
81-224	NORTH WESTERN	11	30km E. of Hwy633	600	550	550	500	500	500
82-04	SOUTH WESTERN	21	5km S. of Thamesville	1900	2100	2100	2100	2100	2100
82-19	CENTRAL	649	10km N. of Bobcaygeon	1000	1000	1000	950	950	950
83-43	CENTRAL	QEW	Niag. Falls	26,200	26,000	28,200	30,000	32,150	33,400

Table 2: Recovered Asphalt Cement Properties of RHM

Contract	RAP		RHM	
	Pen @ 25°C (0.1mm)	K. Visc. @ 135°C (mm^2/sec)	Pen @ 25°C (0.1mm)	K. Visc. @ 135°C (mm^2/sec)
81-74	35	1073	53	457
81-224	31	517	57	326
82-04	28	508	36	498
82-19	20	1456	63	401
83-43	22	1167	30*	708*

* *Recovered one year after construction*

Table 3: Aggregate Description for Various Mixes

Mix Identification	Coarse Aggregate	Fine Aggregate
81-74	Crushed gravel containing Palaeozoic carbonates and marble granite	Silica rich natural sand
81-224	Crushed igneous gravel with mainly granite and granite gneiss	Silica rich natural sand
82-04	Crushed carbonate gravel with < 10% granite and gneiss	Silica rich natural sand
82-19	Crushed limestone gravel	Carbonate & silicate natural sand
83-43	Steel slag (DOFASCO)	Steel slag (DOFASCO)
HL-4	95% carbonate rock, 5% granite	95% carbonate rock, 5% granite

Table 4: Mix Design Properties of Recycled Hot Mixes and HL4 Mix

CONTRACT NUMBER (Mix Type)	RECYCLING RATIO	ASPHALT CEMENT, %			Stability N	Flow 25mm	VMA %	Ret., % 4.75mm	Air Voids %
		ADDED	TOTAL	PEN					
81-224 (HL4)	60/40	2.7	5.2	300/400	10,000 (16,887)	9.2 (11.4)	N/A (14.6)	40	2.8 (1.4)
82-19 (HL4)	30/70	4.5	5.3	150/200	10,000 (12,080)	13.5 (14.2)	12.5 (14.3)	45	1.6 (1.0)
81-74 (HL4)	50/50	2.3*	5.0	300/400	16,760 (14,469)	12.5 (12.1)	N/A (14.8)	45	2.8 (2.8)
82-04 (HL4)	70/30	1.7	5.3	150/200	16,280 (17,658)	13.8 (13.5)	15.6 (15.3)	36	3.0 (1.9)
83-43 (HL1)	25/75	3.6	5.3	85/100	23,000 (24,042)	18.1 (17.4)	11.8 (9.5)	45	2.7 (2.6)
(HL4)	-	-	5.3	85/100	13600	10.4	13.8	45	4.0

** 0.5% antistripping agent added*

Plant check results are given in parentheses

Table 5: Aggregate Gradations for HL4 and RHM Mixes

Mix		% Retained											
		Sieve size, mm								Sieve size, μm			
		19	16	13.2	9.5	6.7	4.75	2.36	1.18	600	300	150	75
HL4	CA:			23.2	68.5	89.0	98.1	98.3	98.5	98.7	98.8	99.0	99.2
	FA:						1.0	20.1	39.0	58.6	82.4	92.4	96.7
	Final:			10.5	31.0	40.3	45.0	55.5	65.9	78.7	89.8	95.4	97.8
81-74	RAP:			3.2	13.4	26.0	38.8	50.7	60.9	71.5	84.1	91.0	94.8
	CA:		4.3	20.4	63.7	88.3	92.7	93.6	94.5	95.4	96.2	97.1	98.0
	FA:						0.4	1.4	5.3	21.0	60.6	88.8	95.9
	Final:		1.3	7.5	25.1	38.4	45.0	52.2	58.3	67.2	82.4	92.3	96.0
81-224	RAP:		1.3	5.8	24.0	38.9	49.0	62.4	72.9	81.2	87.6	91.7	94.9
	CA:		2.8	17.2	55.7	81.6	94.6	95.2	95.7	96.3	96.9	97.4	98.0
	FA:					1.0	9.1	29.4	51.3	72.2	87.8	95.1	97.9
	BLSA:	0.5	1.0	1.6	3.6	5.7	8.3	17.1	32.2	55.8	85.3	97.5	99.4
	Final:		1.0	4.9	19.6	31.2	40.0	52.6	64.9	77.0	88.1	93.7	96.4
82-04	RAP:	2.0	4.0	7.0	18.0	33.0	44.2	57.0	64.0	71.0	84.0	93.0	95.5
	FA: CA:		0.3	8.5	59.9	88.7	98.1	98.3	98.6	98.8	98.0	89.3	99.5
					0.5	1.0	2.0	4.1	8.0	14.5	33.9	81.7	94.2
	Final:	1.4	2.8	5.3	15.6	27.5	36.0	45.4	51.2	57.8	71.8	90.4	95.4
82-19	RAP:	1.3	6.7	10.0	23.0	34.7	45.0	59.7	70.3	79.0	87.0	93.0	96.0
	CA:		3.5	19.5	63.4	88.5	97.8	97.9	98.2	98.5	98.8	99.1	99.0
	FA:						1.9	16.4	35.1	51.5	70.2	83.1	91.1
	BLSA:				1.4	2.9	5.3	12.3	24.1	39.8	65.6	88.5	96.1
	Final:	0.4	3.1	9.0	26.8	38.3	45.0	53.7	62.9	71.9	83.1	92.1	96.2
83-43	RAP:			1.0	8.0	22.0	32.0	46.0	58.0	68.0	80.0	89.0	92.8
	CA:			2.0	34.6	89.7	94.5	95.2	95.9	96.7	97.4	98.1	98.8
	FA:							39.1	62.4	75.4	85.0	91.1	95.2
	Final:			1.0	15.6	32.8	45.0	62.9	74.6	82.2	88.7	93.4	96.0

Table 6: Experimental Design for Direct Tension Tests

Mix type		Temperature°C		
		21	-5	-35
81-74	Plant	2	2	2
	Lab	2	2	2
81-224	Plant	2	2	2
	Lab	2	2	2
82-04	Plant	2	2	2
	Lab	2	2	2
82-19	Plant	2	2	2
	Lab	2	2	2
83-43	Plant	2	2	2
	Lab	-	-	-
HL4	Lab	2	2	2

Note: Two samples per cell

Table 7: Failure Strains ($x10^{-3}$ mm/mm) and Fracture Temperature of Various Mixes

Mix	Test Temperature °C			Fracture Temperature °C
	21	-5	-35	
HL4	7.545	2.630	0.098	-23
81-74P	3.635	1.555	0.152	-19
81-74L	2.975	1.056	0.095	-13
81-224P	4.310	1.979	0.103	-23
81-224L	3.820	1.574	0.160	-17
82-04P	2.315	0.842	0.090	-13
82-04L	1.440	0.698	0.148	-13
82-19P	5.695	2.615	0.079	-26
82-19L	4.958	2.050	0.107	-24
83-43P	1.465	0.603	0.212	-7

Table 8: Stiffness Modulus Values (MPa)

Mix	@ Test Temperature			% Difference, Plant/Lab		@ Fracture Temperature		
	21°C	-5°C	-35°C	21°C	-35°C	Estimated from graph (Figs.1&2)	McLeod's Maximum	Estimated / Mcleod's
HL4	36	460.0	20618	-	-	5000	2760	1.8
81-74P	268.0	797.0	24635	-21	-30	3800	1800	2.1
81-74L	338.5	1275.5	35280			2800	790	3.5
81-224P	157.5	637.2	34341	-35	38	6100	2760	2.2
81-224L	243.0	938.0	24920			2900	1100	2.6
82-04P	754.0	2761.5	53940	-54	142	5400	790	6.8
82-04L	1631.5	3258.5	22231			5400	790	6.8
82-19P	148.4	480.6	38645	-37	20	9000	3200	2.8
82-19L	237.0	923.0	32315			7800	3050	2.6
83-43P	1151.0	3032.0	24185	-	-	3600	-	-

Table 9: Thermally Induced Strains ($x10^{-3}$ mm/mm) for Plant Mixes

Temperature °C	HL4	Contract				
		81-74	81-224	82-04	82-19	83-43
-38	1.22	0.986	0.7	0.91	0.71	1.0
-29	1.12	0.796	0.7	0.815	0.71	0.8
-7	0.51	0.896	0.335	0.509	0.339	0.635
4	0.20	0.497	0.335	0.3	0.271	0.440

Table 10: Thermally Induced Strains ($x10^{-3}$ mm/mm) for Laboratory Mixes

Temperature (°C)	Contract			
	81-74	81-224	82-04	82-19
-38	1.479	1.49	0.889	0.996
-8	-	-	0.49	0.49
-5	.39	.49	-	-

Table 11: Condition Survey Data

Contract	Cracks, m/km lane (Three years after construction)					
	Type A	Type B	Type C	Type D	Type T	Total
81-74	53	249	97	18	0	417
81-224	30	168	127	0	0	325
82-04	198	2068	492	151	0	2909
82-19	65	54	0	0	0	119
83-43 *	34	31	5	0	2896	2966

Note: *A = Hairline crack*
B < 5mm
C =5-10mm
D =10-20mm
T = Treated

* = Two years after construction

Table 12: Cracking Resistance Rating of Different Mixes Based on Fracture Temperature, Stiffness at -35°C and the Field Data

Cracking Resistance Ranking	Mix (by Contract Number)				
	Fracture Temperature		Stiffness at -35°C		Field Data
	Plant	Lab	Plant	Lab	
1 (Very High)	82-19	HL4 82-19	81-74	HL4 82-04	82-19
2 (High)	81-224	81-224	81-224	81-224	81-224
3 (Medium)	81-74	81-74 82-04	82-19	82-19	81-74
4 (Poor)	82-04 83-43	-	82-04	81-74	82-04 83-43

Fig. 1: Stiffness Gradients for Plant Mixes

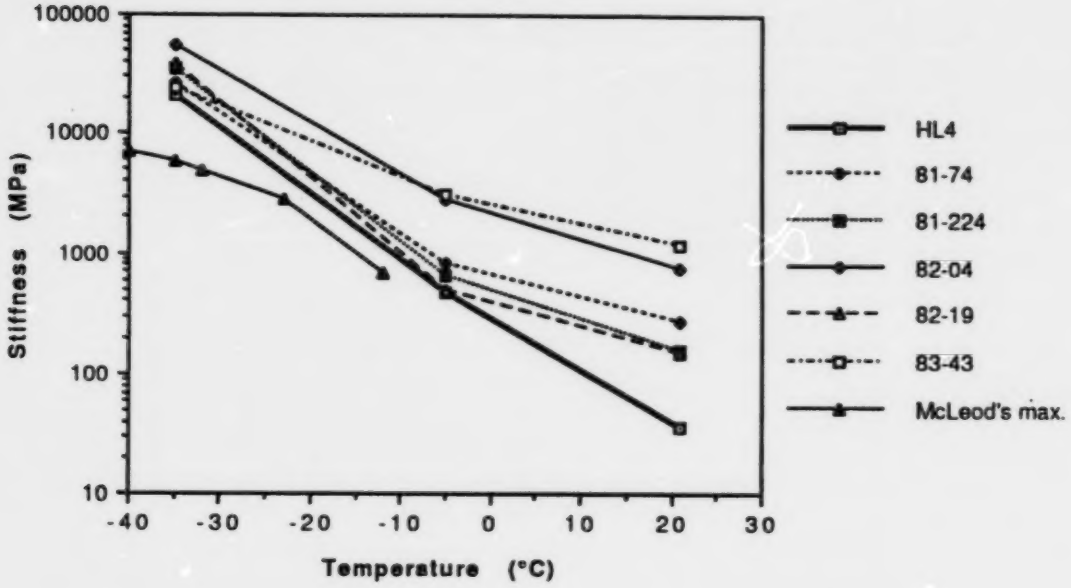


Fig. 2: Stiffness Gradients for Laboratory Mixes

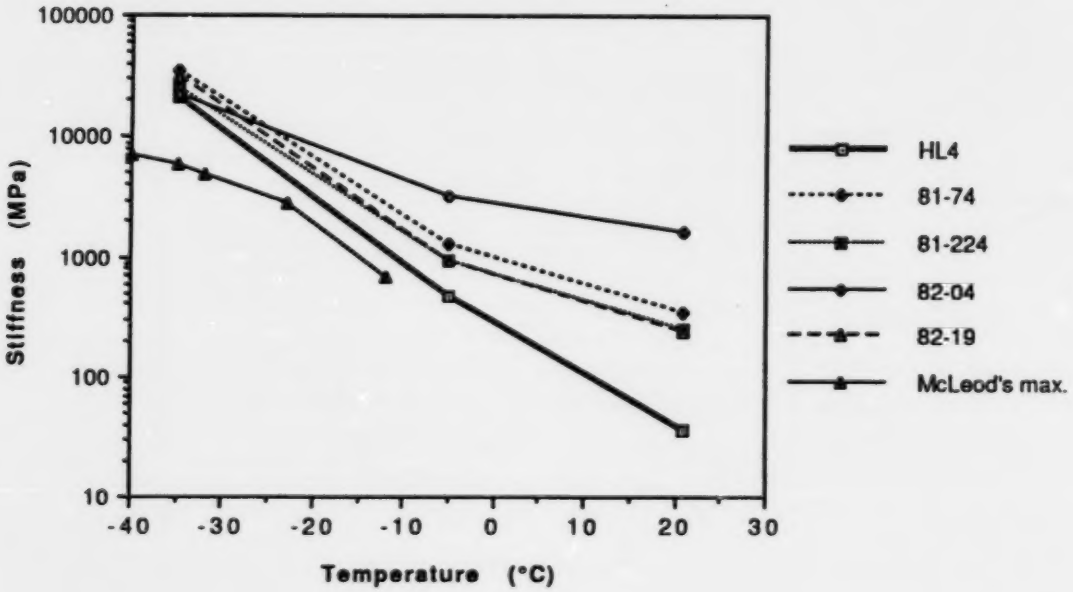


Fig. 3: Comparison of Plant and Laboratory Mix Stiffnesses at Different Temperatures

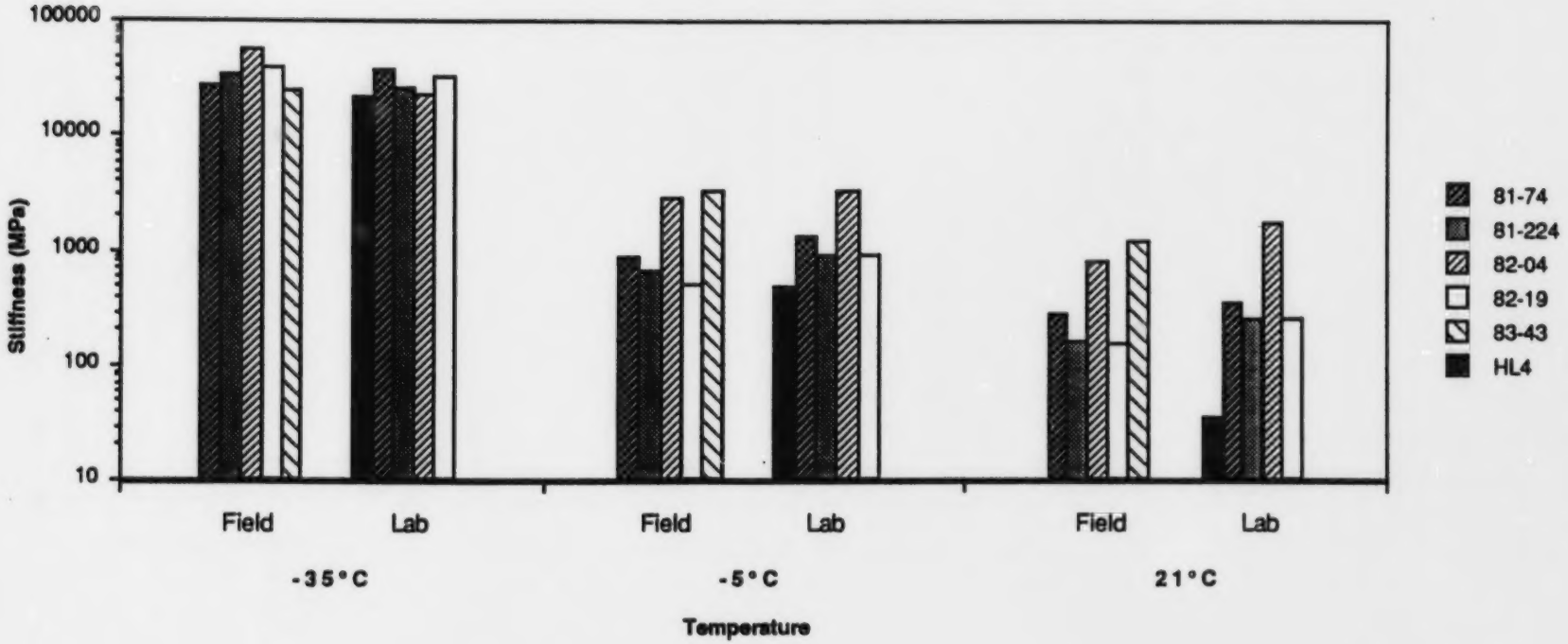


Fig. 4: Relationship of Laboratory and Plant Mix Stiffnesses at -35°C, -5°C, and 21°C

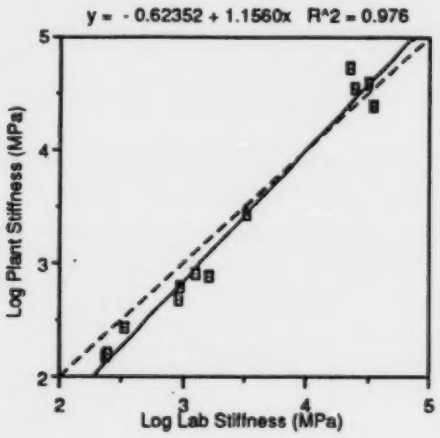


Fig. 5: Estimation of Fracture Temperature for Plant Mixes

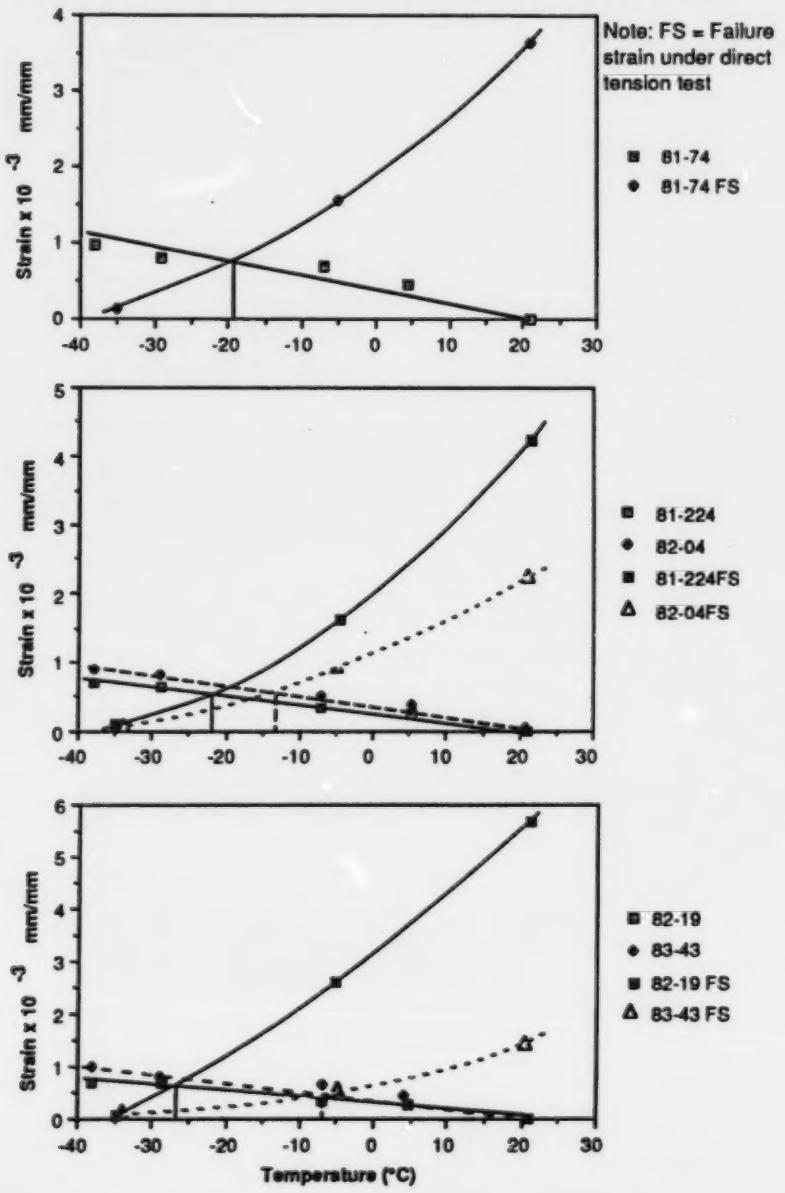


Fig. 6: Estimation of Fracture Temperature for Laboratory Mixes

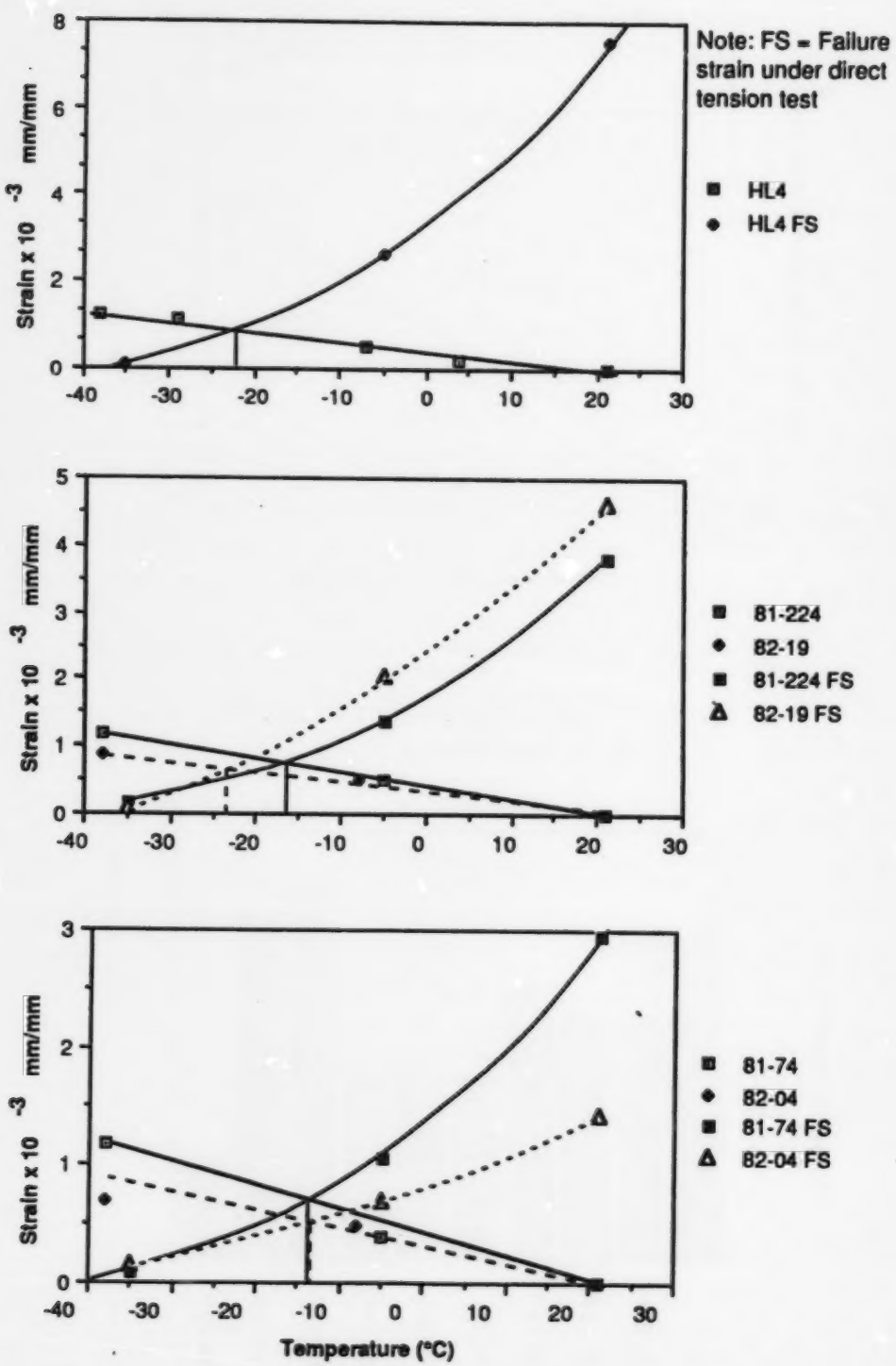


Fig. 7: Comparison of Plant and Lab Mix Fracture Temperatures

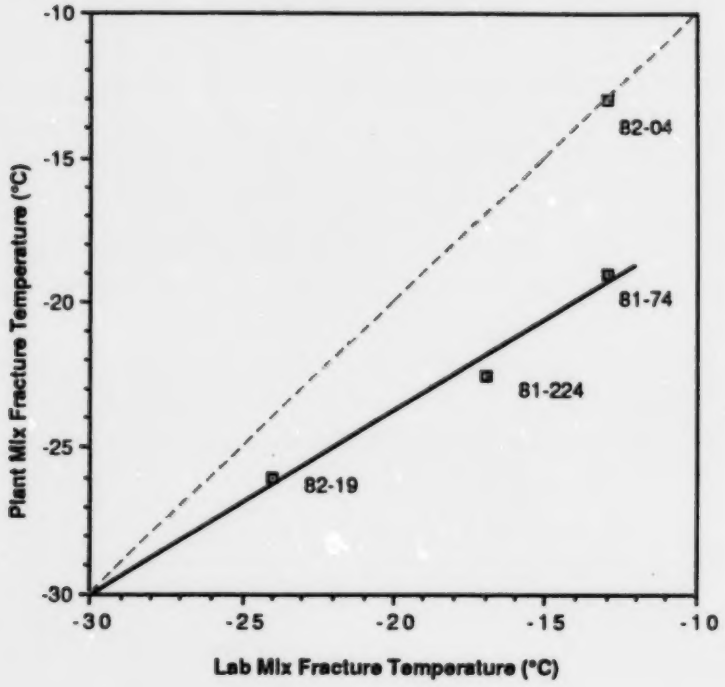


Fig. 8: Fracture Temperature Comparison of Various Asphalt Mixes

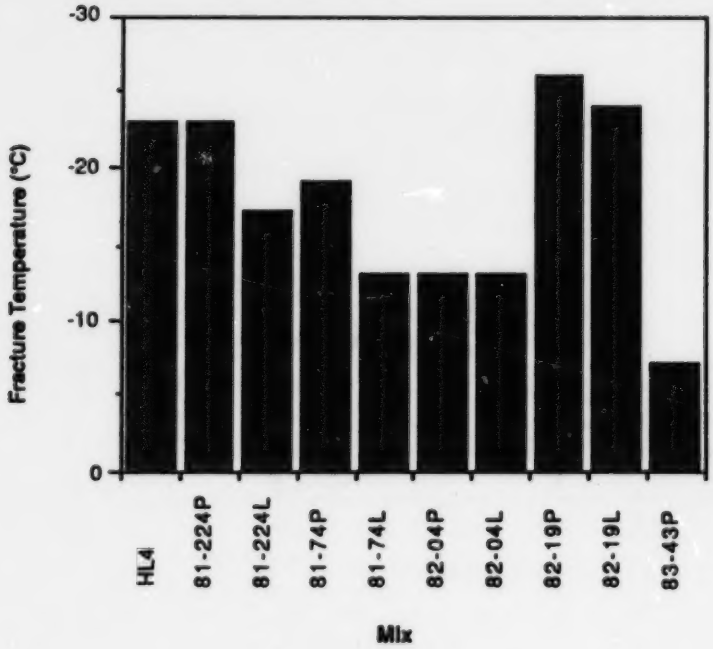
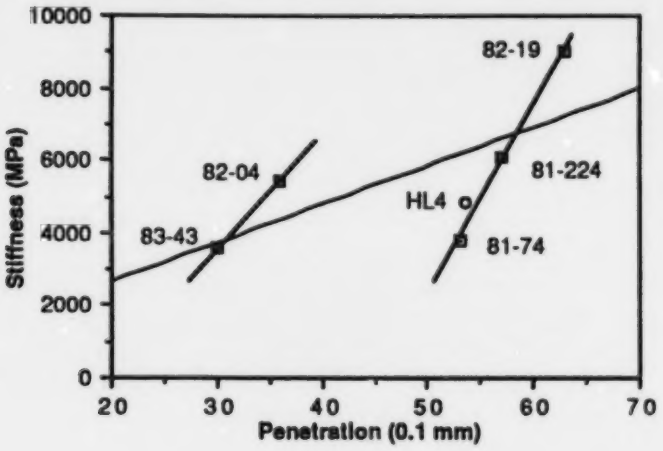
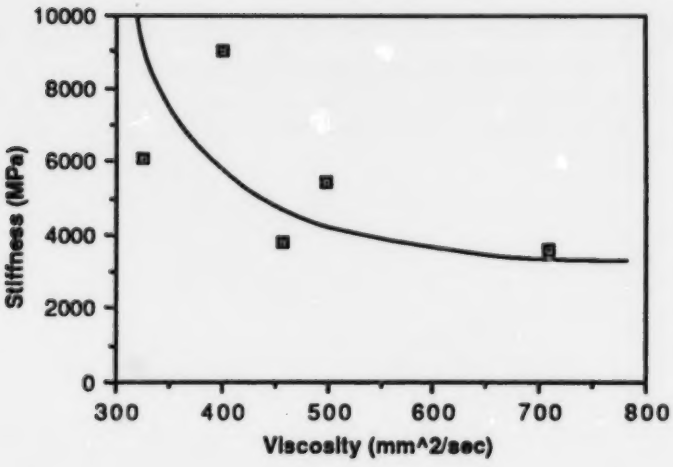


Fig. 9: Plant Stiffness at Fracture Temperature vs Penetration and Kinematic Viscosity



(a)



(b)

Fig. 10a: Plant Mix Fracture Temperature vs RHM Penetration

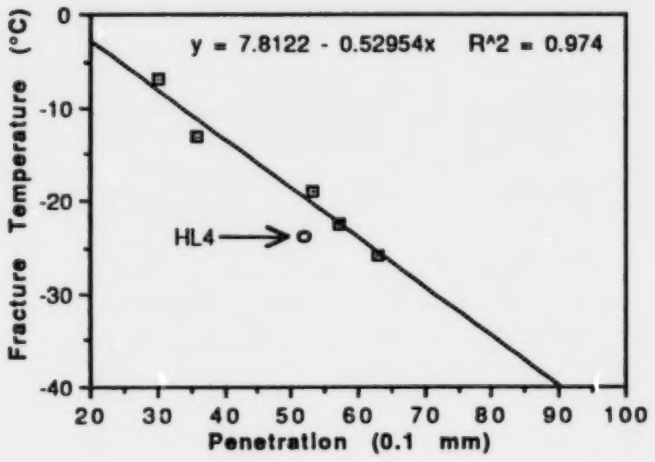


Fig. 10b: Plant Mix Fracture Temperature vs RHM Kinematic Viscosity

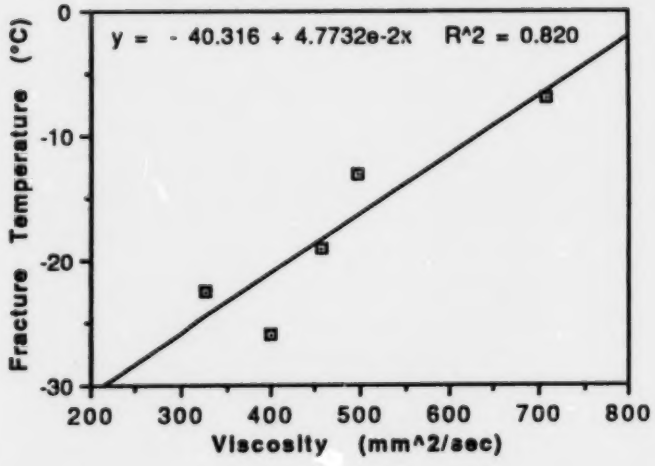


Fig. 11a: Total Length of Cracks versus Fracture Temperature

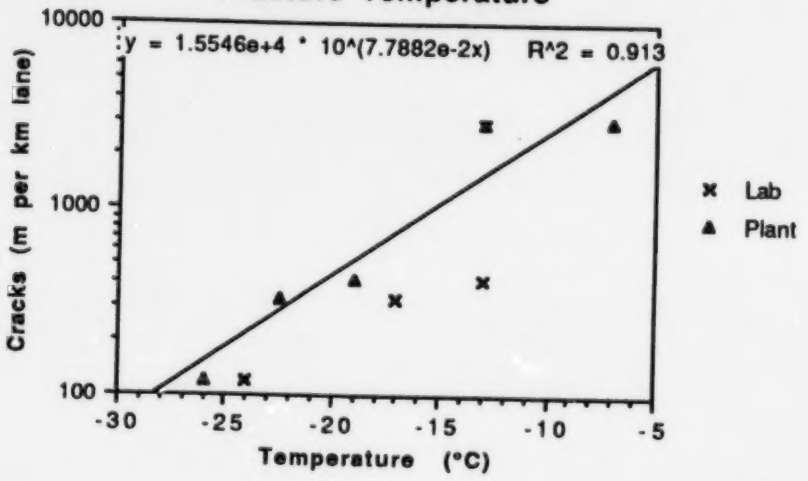


Fig. 11b: Total Length of Cracks versus Stiffness at Fracture Temperature

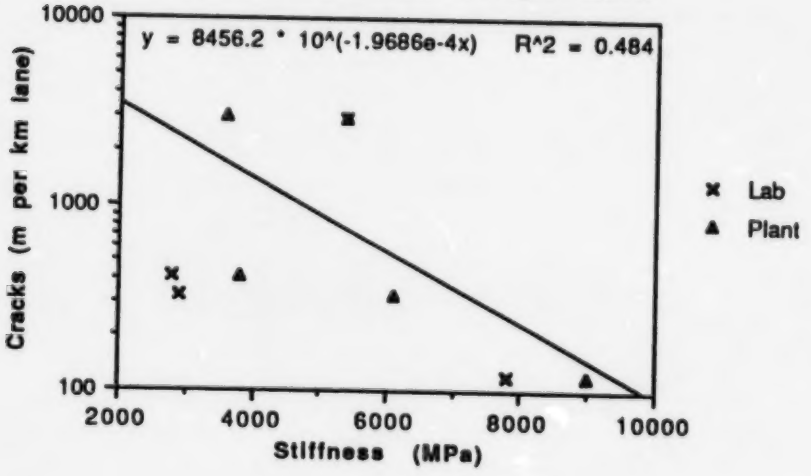


Fig. 12a: Total Length of Cracks versus RHM Penetration

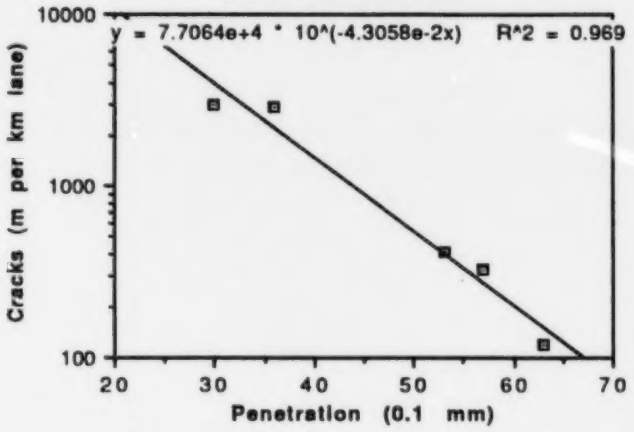


Fig. 12b: Total Length of Cracks versus RHM Kinematic Viscosity

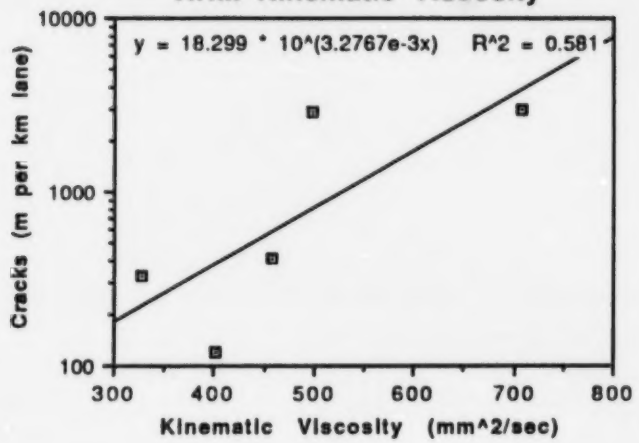


Fig. 13: Stiffness Modulus at Various Fracture Temperatures